



Bulletin 95



Fibre Reinforced Concrete: From Design to Structural Applications

Proceedings of the ACI-*fib*-RILEM International
Workshop - FRC2018

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Technical Report

Proceedings of the ACI-*fib*-RILEM
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Any publication not having met the above requirements will be clearly identified as a preliminary draft.

fib Bulletin 95 is published as a technical report and is a collection of contributions to a workshop that was co-sponsored by the *fib* and the American Concrete Institute (ACI). The authors have presented their individual views. Although these contributions have not been discussed in any of the *fib*'s working bodies, the subject matter is highly topical and believed to be of general interest to members of the *fib*.

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Foreword

This bulletin presents a compilation of the papers presented at the 3rd International workshop on Fibre-Reinforced Concrete, "FRC2018, from Design to Structural Applications". The workshop was a joint ACI - Rilem - *fib* event, which expresses inter-federation collaboration and broad expertise.

The aim of the workshop was to provide state-of-the-art information on recent progress in term of specifications development, actual applications, and to expose users and researchers to tomorrow's challenges in the design and construction of a wide variety of structural applications. The topic is thus fitting and is a clear supplement to the work of *fib* TG4.1 "Fibre-reinforced concrete". The aim of TG4.1 is to make theoretical and practical developments in the field of fibre-reinforced concrete materials technology, rheology and materials characterisation and modelling and to present these developments in code-type formulations.

Recently, it was decided to merge the work of TG4.1 and TG4.2 "Ultrahigh performance fibre reinforced concrete" to prepare one joint bulletin. At the time of writing, this bulletin is being finalised. Also, a framework bulletin for the *fib* Model Code MC2020 will be prepared.

I'm very grateful to Fausto Minelli (University of Brescia, Italy), for making the colossal effort to convert this collection of papers into an *fib* bulletin. And many thanks to Corinne Bottollier-Dépois for formatting this bulletin.

Tor Arne Martius-Hammer

Deputy chair of Commission 4: *Concrete and Concrete Technology*

Preface

The first international FRC workshop supported by RILEM and ACI was held in Bergamo (Italy) in 2004. At that time, a lack of specific building codes and standards was identified as the main inhibitor to the application of this technology in engineering practice. The workshop aim was placed on the identification of applications, guidelines, and research needs in order for this advanced technology to be transferred to professional practice.

The second international FRC workshop, held in Montreal (Canada) in 2014, was the first ACI-*fib* joint technical event. Many of the objectives identified in 2004 had been achieved by various groups of researchers who shared a common interest in extending the application of FRC materials into the realm of structural engineering and design. The aim of the workshop was to provide the State-of-the-Art on the recent progress that had been made in term of specifications and actual applications for buildings, underground structures, and bridge projects worldwide.

The rapid development of codes, the introduction of new materials and the growing interest of the construction industry suggested presenting this forum at closer intervals. In this context, the third international FRC workshop was held in Desenzano (Italy), four years after Montreal. In this first ACI-*fib*-RILEM joint technical event, the maturity gained through the recent technological developments and large-scale applications were used to show the acceptability of the concrete design using various fibre compositions.

The growing interests of civil infrastructure owners in ultra-high-performance fibre-reinforced concrete (UHPFRC) and synthetic fibres in structural applications bring new challenges in terms of concrete technology and design recommendations. In such a short period of time, we have witnessed the proliferation of the use of fibres as structural reinforcement in various applications such as industrial floors, elevated slabs, precast tunnel lining sections, foundations, as well as bridge decks. We are now moving towards addressing many durability-based design requirements by the use of fibres, as well as the general serviceability-based design. However, the possibility of having a residual tensile strength after cracking of the concrete matrix requires a new conceptual approach for a proper design of FRC structural elements.

With such a perspective in mind, the aim of FRC2018 workshop was to provide the State-of-the-Art on the recent progress in terms of specifications development, actual applications, and to expose users and researchers to the challenges in the design and construction of a wide variety of structural applications.

Considering that at the time of the first workshop, in 2004, no structural codes were available on FRC, we have to recognize the enormous work done by researchers all over the world, who have presented at many FRC events, and convinced code bodies to include FRC among the reliable alternatives for structural applications. This will allow engineers to increasingly utilize FRC with confidence for designing safe and durable structures.

Many presentations also clearly showed that FRC is a promising material for efficient rehabilitation of existing infrastructure in a broad spectrum of repair applications. These cases range from sustained gravity loads to harsh environmental conditions and seismic applications, which are some of the broadest ranges of applications in Civil Engineering.

The workshop was attended by researchers, designers, owner and government representatives as well as participants from the construction and fibre industries. The presence of people with different expertise provided a unique opportunity to share knowledge and promote collaborative efforts. These interactions are essential for the common goal of making better and sustainable constructions in the near future.

The workshop was attended by about 150 participants coming from 30 countries. Researchers from all the continents participated in the workshop, including 24 Ph.D. students, who brought their enthusiasm in FRC structural applications.

For this reason, the workshop Co-chairs sincerely thank all the enterprises that sponsored this event. They also extend their appreciation for the support provided by the industry over the last 30 years which allowed research centers to study FRC materials and their properties, and develop applications to making its use more routine and accepted throughout the world. Their important contribution has been essential for moving the knowledge base forward.

Finally, we appreciate the enormous support received from all three sponsoring organizations of ACI, *fib* and Rilem and look forward to paving the path for future collaborations in various areas of common interest so that the developmental work and implementation of new specifications and design procedures can be expedited internationally.

June 2018

Bruno Massicotte, Fausto Minelli, Barzin Mobasher, Giovanni Plizzari

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1) MECHANICAL CHARACTERIZATION

Round-Robin Test on Various Test-Methods for Flexural Behavior of Steel Fiber Reinforced Sprayed Concretes

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Abstract

According to the actual European standard EN 14487-1 the potential of steel fiber reinforced sprayed concrete is characterized by flexural strength tests (first peak, ultimate and residual). In most cases, this is performed by a four-point bending test on beam specimens, specified in EN 14488-3. As an alternative test method, a three-point bending test on square panels with notch is recommended by EFNARC. It is argued as main benefit of the latter test method, that the geometry and dimensions of the panels are equal to those of specimens used for measuring the energy absorption capacity according to EN 14488-5. Hence, the specification of the ductility of fiber reinforced concretes according to EN 14487-1 in terms of residual strength and energy absorption capacity can be achieved preparing only one type of specimen. Furthermore, the EFNARC guideline points out a smaller scatter of test results, compared to the beam tests according to EN 14488-3.

Before the EFNARC method will be considered in EN 14487-1, the relevant CEN TC 104/WG10 requests for adequate proofs. These were performed within a round-robin test (RRT) on testing the flexural behavior of steel fiber reinforced sprayed concrete by the standardized EN 14488-3 method as well as by the proposed EFNARC method. The aim of this RRT was to investigate the comparability and correlation between the two test methods. Furthermore, the scatter of both methods was assessed. The RRT has been organized by Ruhr University Bochum, in whose labs the steel fiber reinforced sprayed concrete specimens (beams according to EN 14488-3 and square panels according to EFNARC guideline) were produced using a robotic spraying machine. The specimens were then tested in five independent laboratories Europe-wide. The results of this RRT are presented in detail. With regard to the residual strength, the relevant material parameter for steel fiber reinforced sprayed concrete, a tendency towards a slightly lower scatter was detected for the EFNARC test method.

Keywords

Sprayed concrete, steel fibers, round-robin test, flexural behavior

1 Introduction

The specification of the ductility of fiber reinforced sprayed concrete according to EN 14487-1 is ascertained by the classification of its residual strengths and/or energy absorptions capacity. In most cases, the residual strengths will be determined if the concrete characteristics are used in a structural design model. According to EN 14487-1, the determination of flexural strengths has to be performed by a four-point bending test on beam specimens specified in EN 14488-3. The residual strengths are classified by a minimum strength level defined for specific intervals of deflection according to EN 14487-1.

With regard to the main field of application of fiber reinforced concrete (i.e. rock stabilization in tunneling) further information concerning the concrete's energy absorption capacity can be essential. The energy absorption capacity has to be determined by performing a bending test on square panels, which are supported on their four edges and loaded centrically with a point load. This test method is specified in EN 14488-5 and was developed by the “National Rail-way Company (SNCF)” in order to simulate the typical loading scenario in terms of biaxial bending. By analogy to the classification of residual strengths, the energy absorption classes are also defined in EN 14487-1.

In EN 14487-1 it is explicitly pointed out that a direct comparison between the two different methods for the specification of ductility by means of residual strengths and energy absorption capacity is not feasible. Depending on project-specific requirements related to the quality of the concrete it could be necessary to perform both test methods. Consequently, in this case, two different types of specimens have to be prepared and tested under different testing conditions.

As an alternative test method to the four-point bending test on beams according to EN 14488-3, a three-point bending test on square panels with notch is recommended by EFNARC. The respective guideline was published in 2011. In this guideline, a series of arguments for performing the EFNARC method instead of the EN 14488-3 method are indicated. As an example, it is argued as main benefit of the EFNARC test method, that the geometry and dimensions of the panels are equal to those of specimens used for measuring the energy absorption capacity according to EN 14488-5. Hence, only one type of specimen has to be prepared for the determination of both material characteristics. Moreover, the EFNARC guideline points out a smaller scatter of test results compared to the beam tests according to EN 14488-3. However, if the scatter of results within the EFNARC test method is lower compared to the EN 14488-3 method remains to be examined, particular as in both guidelines it is noted that there is a lack of information in terms of precision. Even though it is a matter of fact, before the EFNARC test method will be considered in EN 14487-1, the relevant CEN TC 104/WG10 requests for adequate proofs. These were performed within a round-robin test (RRT) on testing the flexural behavior of steel fiber reinforced sprayed concrete by the standardized EN 14488-3 method as well as by the proposed EFNARC method. The aim of this RRT, that was organized and coordinated by the Institute for Building Materials of the Ruhr University Bochum, was to investigate the comparability and correlation between the two test methods. Furthermore, the scatter of both methods was assessed.

2 Test methods

The flexural strengths (first peak, ultimate and residual) according to EN 14488-3 are determined by testing beam specimens with dimensions of 75 (height) $\times$ 125 (width) $\times$ 500 (min. length) mm³, cut from sprayed panels. The beams are tested in a four-point bending test setup (Figure 1a), whereby the total span length (distance between supporting rollers) is 450 mm and the distance between the loading rollers is 150 mm. The applied load is measured as a function of deflection.

According to EFNARC, the flexural strengths (limit of proportionality, ultimate and residual) are determined by performing a three-point bending test on notched panel specimens with dimensions of 600 (width) $\times$ 600 (length) $\times$ 100 (height) mm³. The span length is 500 mm (Figure 1b). The notch depth is 10 mm and correspondingly, the effective cross section in the mid-span (h_{sp}) is 90 mm. The panels should be prepared by levelling its surface at a panel thickness of 100 mm immediately after spraying. By analogy to EN 14651, the test method for metallic fiber concrete, the load-crack mouth displacement (CMOD) curve is recorded.

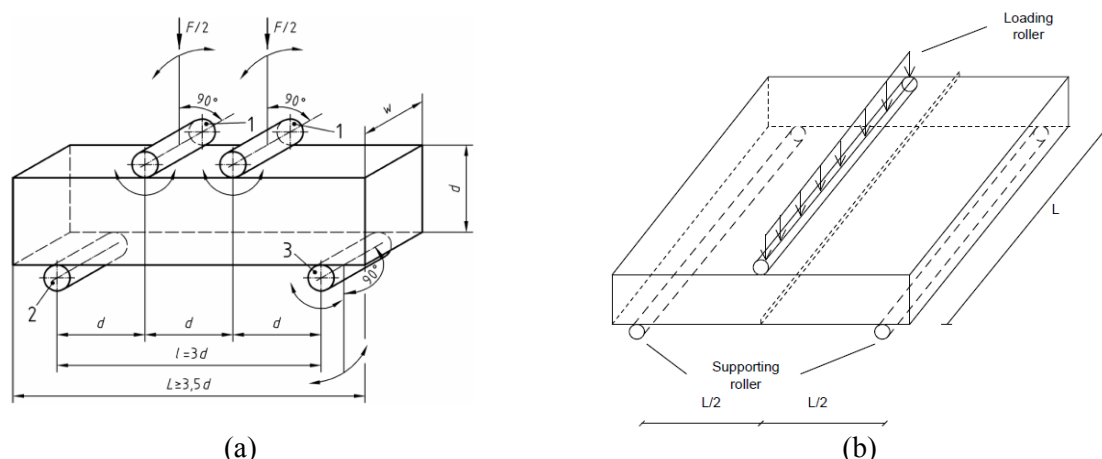


Figure 1: Test setup according to EN 14488-3 (a) and according to EFNARC (b)

For both test methods, the ultimate flexural strength f_{ult} is calculated from the maximum load P_{ult}/F_{ult} . According to EN 14488-3, the first peak strength f_{fp} is calculated from the first peak load P_{a1} , corresponding to first crack formation, as shown in Figure 2a. In consistency with the residual strength classes defined in EN 14487-1, the residual strengths (f_{r1} , f_{r2} , f_{r4}) are calculated from the minimum loads (P_{r1} , P_{r2} , P_{r4}) of the load-deflection curves that lie within the intervals D1 (0,5 mm – 1 mm), D2 (0,5 mm – 2 mm) and D3 (0,5 mm – 4 mm) (Figure 2b).

In case of the EFNARC test method, the LOP $f_{ct,L}$ is calculated from the maximum load F_L recorded in the CMOD interval between 0 mm and 0.05 mm (Figure 3a). As illustrated in Figure 3b, the residual strengths (f_{r1} , f_{r2} , f_{r3} , f_{r4}) are calculated from the loads F_j ($j=1, 2, 3, 4$) corresponding with $CMOD_j$ ($j=1, 2, 3, 4$). Note that the classification of the residual strengths according to EN 14487-1 is not feasible, because the residual strengths classes are not conform to the definition of the residual strengths according to EFNARC.

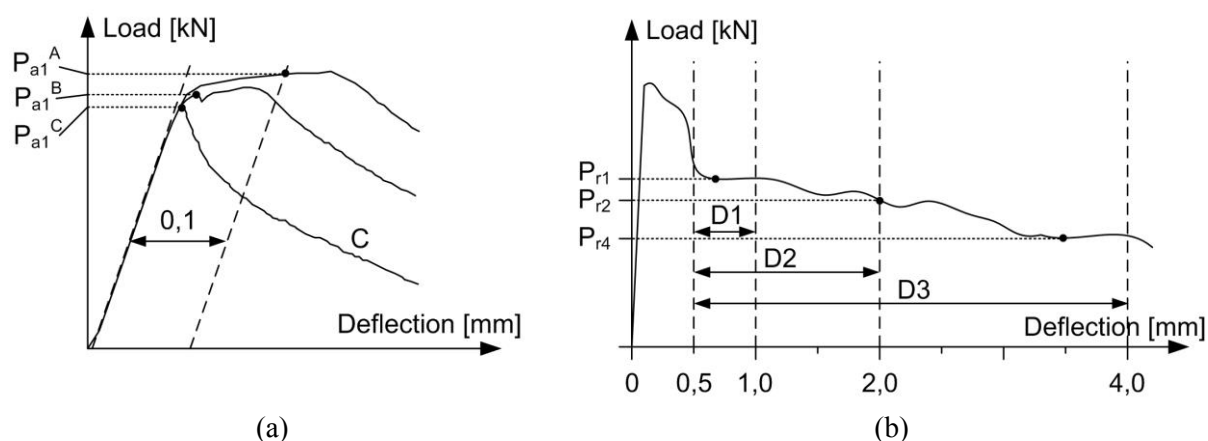


Figure 2: First peak load P_{fp} (a) and loads P_{ri} for the determination of residual strength (b) according to EN 14488-3

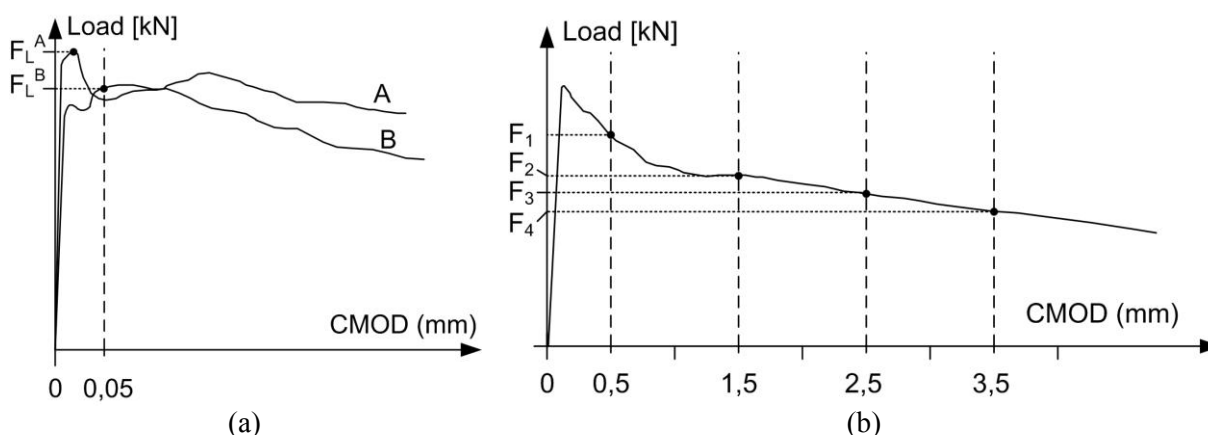


Figure 3: Load F_L for the determination of the LOP (a) and load F_j for the determination of residual strengths according to EFNARC

3 Round-Robin Test (RRT)

3.1 General

In order to investigate the comparability and correlation between the two test methods and to determine the scatter, a round-robin test (RRT) was conducted within the scope of TC 104/WG10. The RRT has been organized by the Institute for Building Materials, Ruhr University Bochum. In whose labs the steel fiber reinforced sprayed concrete specimens were produced using a robotic spraying machine. The specimens were then tested in five independent laboratories Europe-wide.

3.2 Materials

The test specimens (beams according to EN 14488-3 and panels according to EFNARC) were produced by spraying a wet-mix concrete. The base-mix design of the sprayed concrete aimed at strength class C30/37 is listed in Table 1. An ordinary Portland cement (450 kg/m³ CEM I 42.5 R) was used at a water-cement-ratio of 0.45. The base-mix contained 1600 kg/m³ of aggregates with a maximum grain size of 8 mm. The fiber content was 50 kg/m³ and the

characteristics of the steel fibers used are listed in Table 2. The dosage of accelerator was set to 5.5 % relative to the content of cement, in order to enable the surface leveling immediately after spraying execution, in case of preparing panels (cf. chapter 3.3).

Table 1: Base-mix design of sprayed concrete

Constituents	Unit	Content
Cement (CEM I 42.5 R)	[kg/m ³]	450
Aggregates (sand, gravel) (grading curve ~ B8)	[kg/m ³]	1600
Steel fiber (DRAMIX 3D 65/35 BG)	[kg/m ³]	50
Water	[kg/m ³]	202.5
w/c-ratio	[-]	0.45
Superplasticizer (PCE)	[%]	0.56
Retarder	[%]	0.4
Injection at the nozzle: Accelerator (alkali-free)	[%]	5.5

Table 2: Characteristic of steel fibers

Characteristics	Unit	DRAMIX 3D 65/35 BG
Shape	[-]	end-hooked
Length	[mm]	35
Diameter	[mm]	0.55
Slenderness (l/d)	[-]	65
Tensile strength f_t	[MPa]	1345

3.3 Production and preparation of test specimens

The wet-mix concrete was sprayed by using the robotic spraying machine available at Ruhr-University Bochum in order to simulate practical and realistic conditions, similar to a real tunnel construction site. This robotic spraying machine allows for a minimization of manual influences on the quality of sprayed concrete, which could have a negative influence on the reproducibility of results. Figure 4 illustrates a schematic description of the sprayed concrete testing rig in the laboratory of Ruhr University Bochum.

Ready-mixed concrete (base-mix, see Table 1) was transported from a local concrete plant to the laboratory (RUB). The concrete was pumped to the robot and sprayed through the nozzle (Ø42 mm) onto the wooden baseplate. During spraying execution, the robot followed a programmed pattern of vertical and horizontal movements including nozzle oscillations. The spraying distance was approximately 1.0 m, similar to typical field scenarios. In the case of preparing panels according to EFNARC, the sprayed concrete on the baseplate was leveled to the required thickness of 100 mm by screeding its surface with a steel profile immediately after the spraying execution. According to EFNARC, any subsequent surface treatment (e.g. grinding, saw cutting etc.) of the hardened sprayed concrete is not permitted. For this reason, the dosage of accelerator had to be adjusted to ensure that on the one hand, the solidification process of the concrete has progressed so far that the concrete was stable enough to stuck on the vertical baseplate, and on the other hand, that the screeding of the surface after spraying execution was still possible. In the case of preparing the EN 14488-3 beams, a surface

leveling was not necessary. After spraying (and leveling), the sprayed concrete was covered with wet jute and plastic membrane and cured for 24 hours. Afterwards, the specimens were cut from shotcrete baseplate. The panels were notched by using a saw with a diamond blade. The prepared specimens were then stored in a fog room (relative air humidity: $\geq 95\%$, temperature of $20 \pm 2^\circ\text{C}$) and cured for 7 days. Afterwards, the specimens were stored at a temperature of $20 \pm 2^\circ\text{C}$ and a relative air humidity of $65 \pm 5\%$ until shipping to the participating laboratories.

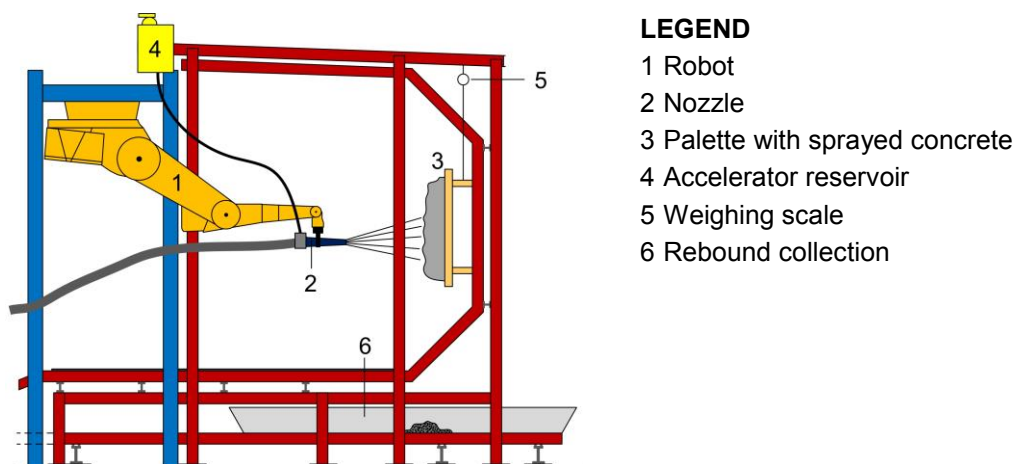


Figure 4: Schematic description of the sprayed concrete testing rig (RUB)

3.4 Flexural tests

Until performing the flexural tests, the specimens were stored under water for one week within the respective laboratories, in order to avoid residual stresses due to shrinkage. In each laboratory 3 beams and 4 panels were tested (In total: 15 beams/20 panels). The allocation of specimens to the respective laboratories was random. The age of the sprayed concrete at the time of testing was 60 ± 2 days. The participating laboratories were obliged to follow the normative specifications as strict as possible. Figure 5 shows the test setup of the 4-point bending test on beams according to EN 14488-3 (a) and the 3-point bending test according to EFNARC (b).



(a)



(b)

Figure 5: Test setup of 4-point bending test on beams according to EN 14488-3 (a) and 3-point bending test on square panels with notch according to EFNARC (b)

4 Results

4.1 General characteristics of fresh and hardened concrete

In order to ensure a good pumpability, a flowable consistency of the ready-mixed concrete was required. This requirement was achieved by adding PCE-superplasticizer in the concrete plant (see Table 1). Depending on the actual transport time of the concrete truck and its discharge time, the consistency, immediately before spraying execution, was between 620 mm and 540 mm (flow table test according to EN 12350-5). The general characteristics of the fresh concrete (i.e. base-mix) are listed in Table 3. The fiber content of the base-mix was measured in a non-destructive way by means of a special test device based on the principle of electromagnetic induction (Breitenbücher and Rahm, 2009). It was between 46 kg/m³ and 53 kg/m³. The targeted strength class C30/37 was proved by conformity check on cube specimens casted with the base-mix according to EN 206 ($f_{cm,cube}=46$ MPa). The mean value of the compressive strength of the sprayed concrete determined on drilling cores taken from the sprayed concrete base plates at the time of testing the flexural strengths was 55 MPa.

Table 3: General characteristics of the fresh concrete (base-mix)

Characteristic	Unit	Test-Method	Mean value
Consistency	[mm]	EN 12350-5	580
Bulk density	[kg/dm ³]	EN 12350-6	2.301
Air void content	[vol.-%]	EN 12350-7	2.1

4.2 Flexural strengths according to EN 14488-3 and EFNARC

In Figure 6 and Figure 7 the stress-deflection and stress-CMOD curves of all specimens (beams according to EN 14488-3: B1-15; panels according to EFNARC: P1-20) tested within the RRT and the respective laboratories (Lab A-E) are illustrated. These stress-displacement curves feature the characteristic shape for steel fiber reinforced concrete under bending load. The specimens are characterized by a linear stress-displacement behavior in the pre-peak branch. It can be seen, that the initial structural behavior of the beams and panels is very similar, respectively. After reaching the first stress peak (first peak strength f_{fp}/LOP $f_{ct,L}$), accompanied by first crack formation, a sudden drop of the curves can be observed. With increasing deformation, the load-bearing capacity persists due to the load transfer of the crack bridging fibers. This ductile post-cracking behavior is characterized by the more or less horizontal shape of the curves in the post-cracking zone. The stress-CMOD curves of the panels clearly show a further stress increase in the post-cracking zone. This increase leads to a second stress maximum in the range between $0.5 \text{ mm} \leq \text{CMOD} \leq 2.0 \text{ mm}$ followed by a gradual slope with increasing deformation. The stress-deflection curves of the beams partially also exhibit a stress increase in the post-cracking zone. Compared to the stress-CMOD curves of the panels, most of the stress-deflection curves of the beams are characterized by an unsteady and jagged development in the post-cracking zone. Hence, an exact range of deformation for the stress maximum in the post-cracking zone cannot be located considering all curves. However, from a global point of view, a gradual slope in the post-cracking zone can be observed, too. The smoother and even more regular development of the stress-CMOD curves compared to the stress-deflection curves is mainly attributed to the fact that more fibers were intersecting the crack area in the case of the panels due to their larger cross-sections.

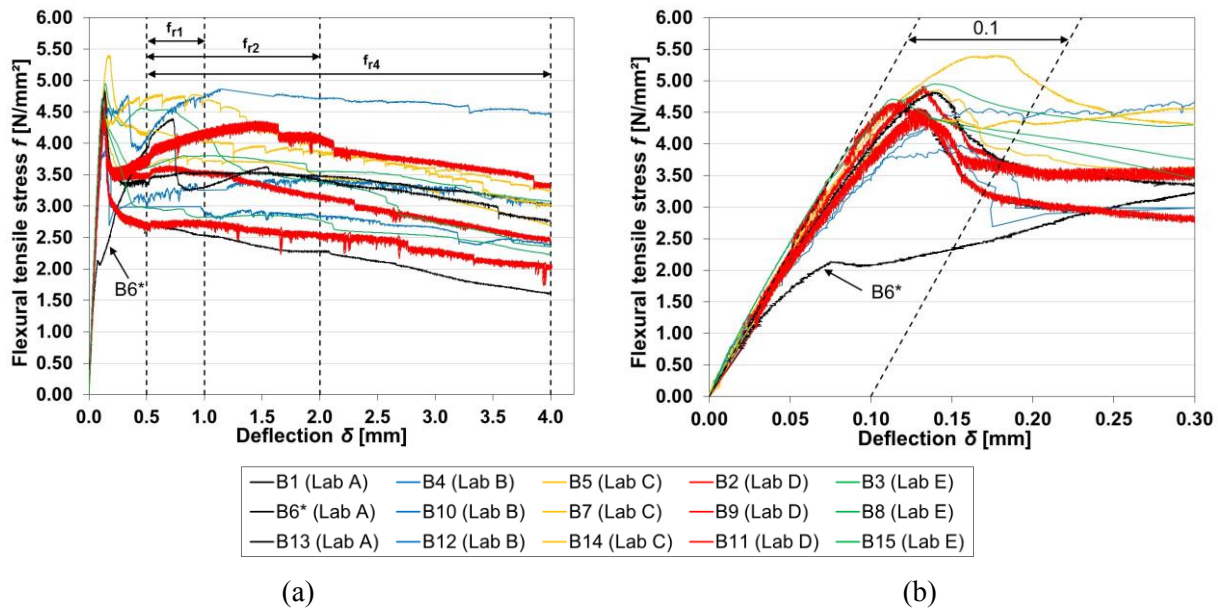


Figure 6: Stress-deflection curves of beam specimens according to EN 14488-3 in the interval of [0 mm – 4.0 mm] (a) and [0 mm – 0.30 mm] (b)

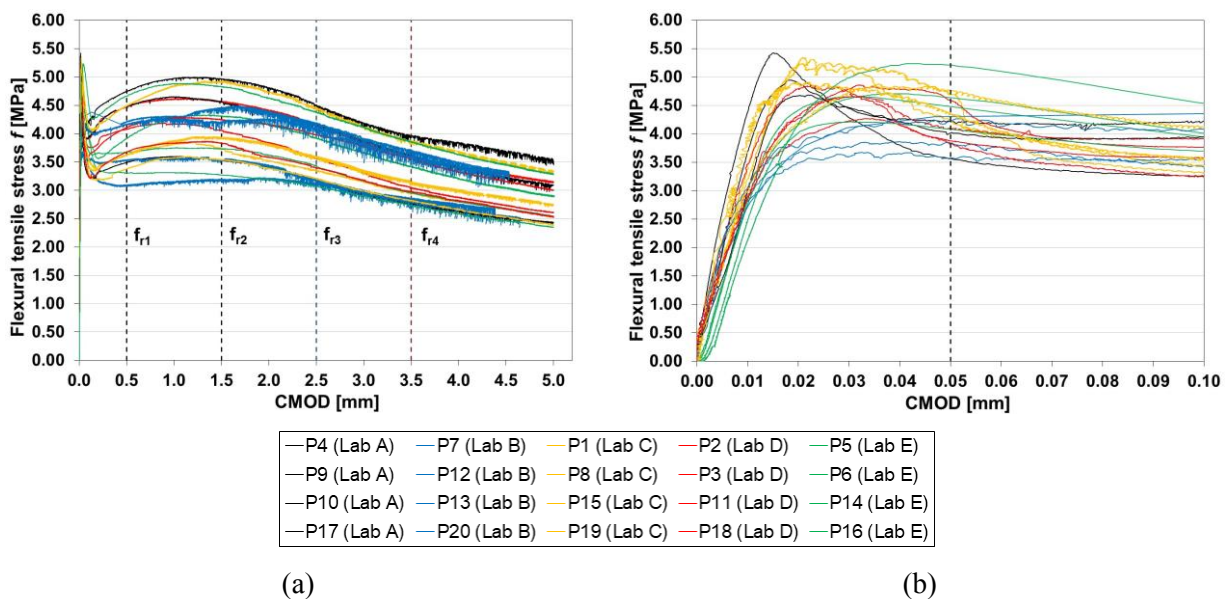


Figure 7: Stress-CMOD curves of panel specimens according to EFNARC in the interval of [0 mm – 5.0 mm] (a) and [0 mm – 0.10 mm] (b)

The flexural tensile strengths (first peak/LOP, ultimate and residual) were calculated from the presented stress-displacement curves of the beams and panels (cf. chapter 2). The total mean values MV, standard deviations s and variation coefficients v were determined and are listed in Table 4 and Table 5. It should be noted, that beam specimen B6* showed an unrepresentative characteristic in the pre-peak branch of the stress-deflection curve (see Figure 6b). It appears that this beam has been pre-damaged, possibly during transport delivery. Hence, beam specimen B6* was excluded from the calculation.

Table 4: Flexural strengths of beam specimens according to EN 14488-3

Flexural strength	Mean value MV [N/mm ²]	Standard deviation s [N/mm ²]	Variation coefficient v [%]
Ultimate strength f_{ult}	4.68	0.33	7.1
First peak strength f_{fp}	4.65	0.34	7.4
Residual strength f_{R1}	3.45	0.62	17.8
Residual strength f_{R2}	3.26	0.52	16.0
Residual strength f_{R4}	2.75	0.61	22.2

Table 5: Flexural strengths of panel specimens according to EFNARC

Flexural strength	Mean value MV [N/mm ²]	Standard deviation s [N/mm ²]	Variation coefficient v [%]
Ultimate strength f_{ult}	4.70	0.48	10.2
LOP $f_{ct,L}$	4.68	0.49	10.4
Residual strength f_{R1}	3.93	0.49	12.5
Residual strength f_{R2}	4.07	0.55	13.4
Residual strength f_{R3}	3.73	0.49	13.2
Residual strength f_{R4}	3.26	0.43	13.0

Interestingly, despite different specimen geometrics and test conditions, the total mean values of the ultimate flexural strengths of the beams ($f_{ult,beams,MV}=4.68$ MPa) and panels ($f_{ult,panels,MV}=4.70$ MPa) are almost identical (cf. Table 4 and Table 5). However, the total scatter of the ultimate flexural strengths is slightly larger for the panels ($s_{panels}(f_{ult})=0.48$ MPa, $v_{panels}(f_{ult})=10.8$ %) compared to the beams ($s_{panels}(f_{ult})=0.33$ MPa, $v_{panels}(f_{ult})=7.1$ %). This could be attributed to the predefined crack location caused by the notch in the case of panel specimens. The absolute as well as statistical mean values of the first peak strengths f_{fp} and LOPs $f_{ct,L}$ differ only marginally from those of the ultimate flexural strengths f_{ult} (cf. Table 4 and Table 5). This is mainly attributed to the subcritical fiber content of 50 kg/m³. As a result, the post-cracking stresses do not exceed the first peak stresses. In some exceptional cases of both test methods, the post-cracking stresses exceed the first peak stresses. However, in these cases, they were only very slightly larger ($\Delta_{max,beams}=f_{ult} - f_{fp}=0.4$ MPa, $\Delta_{max,panels}=f_{ult} - f_{ct,L}=0.22$ MPa). For these cases, it could be expected that the distribution and orientation of steel fibers in the cracked area of the specimens was comparatively favorable.

In Table 4 and Table 5 it can be seen, that the residual strengths determined for the beams ($f_{R1,MV}=3.45$ MPa, $f_{R2,MV}=3.26$ MPa, $f_{R4,MV}=2.75$ MPa) are slightly lower than the residual strengths of the panels ($f_{R1,MV}=3.93$ MPa, $f_{R2,MV}=4.07$ MPa, $f_{R3,MV}=3.73$ MPa, $f_{R4,MV}=3.26$ MPa). The total scatter of the residual strengths is larger compared to the scatter of the ultimate strengths as well as first peak strengths and LOPs, respectively. Comparing the total standard deviations or variation coefficients of residual strengths it can be seen, that the scatter of results for the beams tested according to EN 14488-3 ($0.52 \text{ MPa} \leq s(f_{Ri,beams,MV}) \leq 0.62 \text{ MPa}$, $16.0 \% \leq v(f_{Ri,beams,MV}) \leq 22.2 \%$) were somewhat larger than for panels tested according to EFNARC ($0.43 \text{ MPa} \leq s(f_{Ri,panels,MV}) \leq 0.55 \text{ MPa}$, $12.5 \% \leq v(f_{Ri,panels,MV}) \leq 13.4 \%$). However, it should be noted that a direct comparison between the absolute as well as the statistical values of the residual strengths of the beams and panels is unfeasible due to their different definitions (cf. chapter 2).

5 Concluding remarks

The results of the RRT presented in the present paper allow to draw the main concluding remarks listed in the following.

1. With regard to the total mean values of ultimate strengths as well as first peak strengths and LOPs, the test results showed an almost exact correlation between the test methods. The total scatter of ultimate strengths as well as first peak strengths and LOPs was slightly larger for the EFNARC test method compared to the EN 14488-3 method.
2. The total mean values of residual strengths determined according to EN 14488-3 were slightly lower than these determined according to EFNARC. The total scatter of residual strengths was somewhat larger for the EN 14488-3 method compared to the EFNARC method. However, a direct comparison between the absolute as well as the statistical values of the residual strengths of the beams and panels is unfeasible due to their different definitions.
3. All in all, it can be stated that there are no significant differences concerning the scatter of results. Hence, it is quite conceivable that the EFNARC method would be considered in EN 14487-1. However, the classification of residual strengths according to EN 14487-1 needs to be adapted.
4. For a final evaluation of both test methods, further comparative studies (i.e. larger numbers of test specimens, variation of fiber content and fiber types etc.) are necessary.

6 Acknowledgements

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A new multi-scale hybrid fibre reinforced cement-based composites

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Abstract

Nowadays polyvinyl alcohol (PVA) and steel hybrid fibre reinforced cement-based composites (HFRCC) are studied to resist static, seismic and impact loading. In this research, calcium carbonate (CaCO₃) whisker is introduced to develop a multi-scale HFRCC, with 35mm hooked end steel fibre, 6mm PVA fibre and CaCO₃ whisker. The compressive and flexural behaviors of the new HyFRCC are explored. Compared to the conventional hybrid fibers (steel fibre and PVA fibre), the addition of multi-scale fibres (steel fibre, PVA fibre and CaCO₃ whisker) can improve the compressive strength and toughness of cement-based composite. Multi-scale fibre usage leads to notable increase to both the flexural strength and flexural toughness values of cement-based composite. Based on these test results, we can conclude that there is a remarkable fiber synergy in multi-scale hybrid fibre reinforced cement-based composites with 35mm hooked end steel fibre, 6mm PVA fibre and CaCO₃ whisker. Moreover, it seems possible that the steel fibres and PVA fibres can be partly replaced by CaCO₃ whiskers, which is very beneficial in decreasing the production cost of HFRCC for potential structural applications.

Keywords

Multi-scale hybrid fibre, CaCO₃ whiskers, compressive behavior, flexural behavior

1 Introduction

Conventional cement-based composites are very brittle materials and the failure of cement-based composites is a progressive and multilevel process, pre-existing or newborn micro-cracks grow to meso-cracks and macro-cracks. Eventually, the macro-cracks lead to the fracture (Banthia & Soleimani, 2005; Nguyen et al., 2012; Bentz, 2000). In this sense, combination of fibres with different sizes and constitutions is generally drawn into cement-based composites to restrict the growth of cracks at different stages in the failure process and improve its mechanical properties. Unfortunately, traditional hybrid fibres can hardly delay micro crack initiation and propagation because of the relatively large length and diameter (Li et al., 2015; Parveen et al., 2013).

For a few years, the authors have worked on the development of a new hybrid fibre reinforced cement-based composites (HFRCC), including steel, polyvinyl alcohol (PVA) fibre and calcium carbonate (CaCO_3) whisker. Compared with conventional HyFRCC (only including steel and PVA fibres) (Soe et al., 2013), the steel and PVA fibres were partly replaced by cheaper CaCO_3 whiskers (approximately \$236 per ton) mainly to improve the mechanical performance as well as reduce the production cost. These researches reveal that the introduction of CaCO_3 whisker can improve mechanical properties of HFRCC effectively by micro-reinforcement and positive fibre synergy with the meso-fibre (PVA fibre) and macro-fibre (steel fibre) (Cao, Zhang, Lv, & Xu, 2014; Cao, Zhang & Lv, 2014; Cao, Zhang & Wei, 2013). The new HFRCC could be used to strengthen and retrofit the bridge and hydraulic structures. Although previous studies presented the optimal steel, PVA fibre and whisker content for mechanical properties and economical efficiency including short steel fibre (13mm length), no information is available about the mechanical properties of this new HFRCC including long steel fibre (35mm length).

It is clearly that the shape and size of fibre have significant impact on the mechanical properties of fibre reinforced cement-based composites. Hence, the objective of this research is studying mechanical properties of this new HFRCC including long hooked end steel fibre (35mm length) in laboratory to evaluate if the excellent static behaviour of this material can be stretched to the HFRCC with longer steel fibre.

2 Experimental program

P.O 42.5R ordinary portland cement (Specific surface area: $356\text{m}^2/\text{kg}$, $45\mu\text{m}$ sieve residue: 14.60%, Dalian Onoda Cement Co. Ltd.), and quartz sand (Fineness modulus: 2.51, media sand, Dalian) were used as the major ingredients in the matrix. A high performance water-reducer (Polycarboxylic acid based, water reducing ratio: 32%, Sika Co. Ltd.) was used to modify the fluid properties of mortars in 0.24–0.53% proportions of cement mass. Chemical constituents of CaCO_3 whisker (Fengzhu Co. Ltd.) are shown in Table 1. Physical properties of steel fibre (Bekaert Co. Ltd.), PVA fibre (Wanwei Co. Ltd.) and whisker are tabulated in Table 2 and their morphologies are shown in Fig.1.

Table 1: Chemical constituents of whisker (wt.%)

CaO	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CO ₂	MgO	SO ₃
54.93	0.29	0.11	0.07	42.07	2.14	0.31

Table 2: Physical properties of fibres and whisker

Fibre type	Length(mm)	Aspect ratio(l/d)	Tensile strength (MPa)	E-modulus (GPa)	Density (kg/m^3)
Steel fibre	35	63.6	$\geq 2\ 000$	210	7800
PVA fibre	6	151.1	1529.5	36.7	1300
Whisker	0.02-0.03	10-60	3 000-6 000	410-710	2860

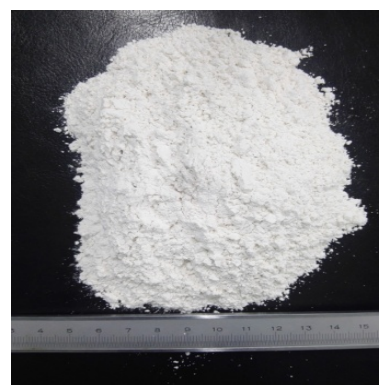
Note: Long steel fibre is hooked end steel fibre.



(a) 35mm hooked end steel fibre



(b) 6mm PVA fibre



(c) CaCO_3 whisker

Figure 1: Morphologies of fibres and whisker

Control specimens, without any fibre and whisker addition, were prepared in the light of the mixture proportioning presented in Table 3. Control specimens were used to assess the influence of fibres and whisker additions. Based on the control specimen proportioning, 5 other mix groups with different type and dosage of steel fibre, PVA fibre and whisker were made as illustrated in Table 4.

Table 3: Mix proportioning of the control specimens (kg/m^3)

Cement	Water	Quartz sand
1220	366	610

Table 4: Content of different fibre in HFRCC.

Group	Fibre type	Volume fraction (%)		
		Steel fibre	PVA fibre	Whisker
Pure	Plain	0	0	0
S2	35mm hooked end steel fibre and 6mm PVA fibre	2	0	0
S1.5P0.5		1.5	0.5	0
S1.5P0.4W1		1.5	0.4	1
S1.25P0.75		1.25	0.75	0
S1.25P0.55W2		1.25	0.55	2

Three prismatic and three cube specimens of each group were casted, all of the results discussed in this research were the average value of the three specimens. Prismatic specimens with dimensions of 100 (width) $\times$ 100(thickness) $\times$ 400(length) mm³ were employed to test flexural behaviour. The cylinder specimens with dimensions of Φ 100 (diameter) $\times$ 200(height) mm³ were employed to test compressive behaviours. For the flexural testing, a computer-controlled electro-hydraulic servo universal equipment of 300kN capacity was employed according to ASTM C1609/C1609M. For the compressive strength testing, a computer-controlled electro-hydraulic servo universal equipment of 2000kN capacity at a displacement rate of 1 mm/min was used according to ASTM C469/C469M.

3 Results and analysis

3.1 Compressive behaviors

The compressive strengths of all mixtures are shown as Figure 2(a). The fibre-reinforced mortars presented a higher compressive strength than that of the pure mortar. The employ of a 2% volume of steel fibre (S2) led an appreciable improvement in compressive strength. The compressive strength value of cement-based composites with steel and PVA hybrid fibres (S1.5P0.5 and S1.25P0.75) were reduced gradually as the PVA fibre amount increased. The introduction of whiskers (S1.5P0.4W1 and S1.25P0.55W2) enhanced the compressive strength of mortar with steel and PVA hybrid fibres and compressive strength of S1.5P0.4W1 was higher than S2, which was caused by the filler effect of whiskers (Cao, Zhang & Wei, 2013).

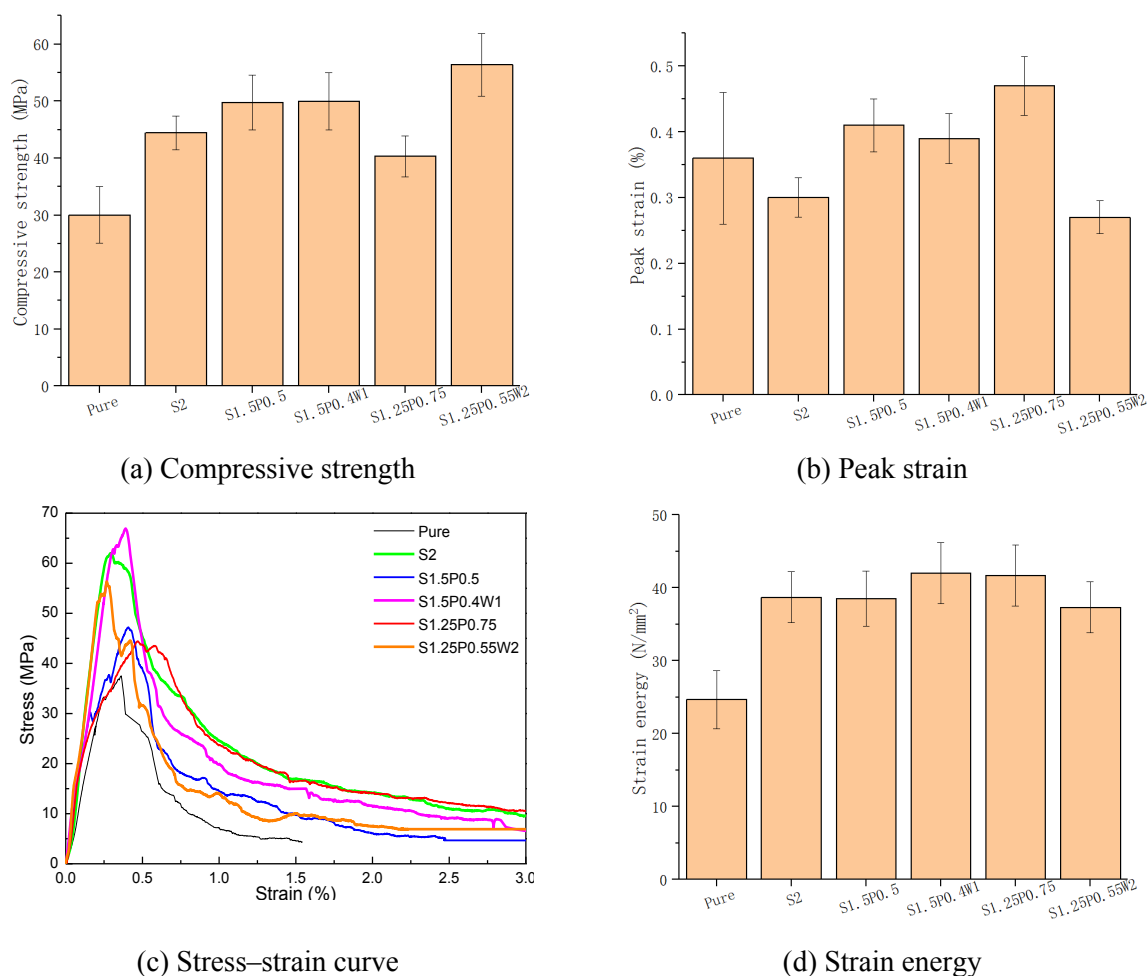


Figure 2: Compressive behaviors of various mixtures

The peak strains of all mixtures are presented in Figure 2(b). Compared to pure mortar, the introduction of steel, PVA hybrid fibre and CaCO_3 whiskers can enhance the peak strain. The maximum value of peak strain reached 0.47% for S1.25P0.75. The stress-strain curves of various composites in uniaxial compression are presented in Figure 2(c). Compared to the control group Pure, the descending or softening branch of the stress-strain curve can be improved by hybrid fibres. Strain energy, which is calculated by the area under the stress-strain curve, is employed to assess the compressive toughness of various mixtures, as shown in Figure 2(d). S2 presented the greatest strain energy, followed by S1.5P0.4W1. The error bars indicate that fibres decrease the data scatters of compressive behaviours of mortar, as shown in Figure 2.

3.2 Flexural behaviors

The flexural strengths for all composites are shown as Figure 3(a). Compared to the control group Pure, all fibre-reinforced mortar presented a significant improvement in flexural strength. Mixture S1.25P0.55W2 presented the greatest flexural strength with an increase more than 130%. Compared to S2, S1.25P0.55W2 still reached nearly 40% of increase in flexural strength. The conventional hybrid fibre-reinforced mortar attained a 5–17% increase in flexural strength, while the multi-scale hybrid fibre-reinforced mortar attained a 13–38%

increase in flexural strength. These reinforcing effects may result from the smaller size and greater length-diameter ratio of PVA fibre and whisker. The multi-scale hybrid fibre system can restrict micro- and meso-cracks in mortar before peak load, hence increase the ultimate flexural tensile stress capacity more efficiently than conventional hybrid fibre system. The compressive and compressive strength in this research are lower than those in previous studies due to bigger specimens and longer steel fiber. (Cao, Zhang, Lv, & Xu, 2014; Cao, Zhang & Lv, 2014; Cao, Zhang & Wei, 2013)

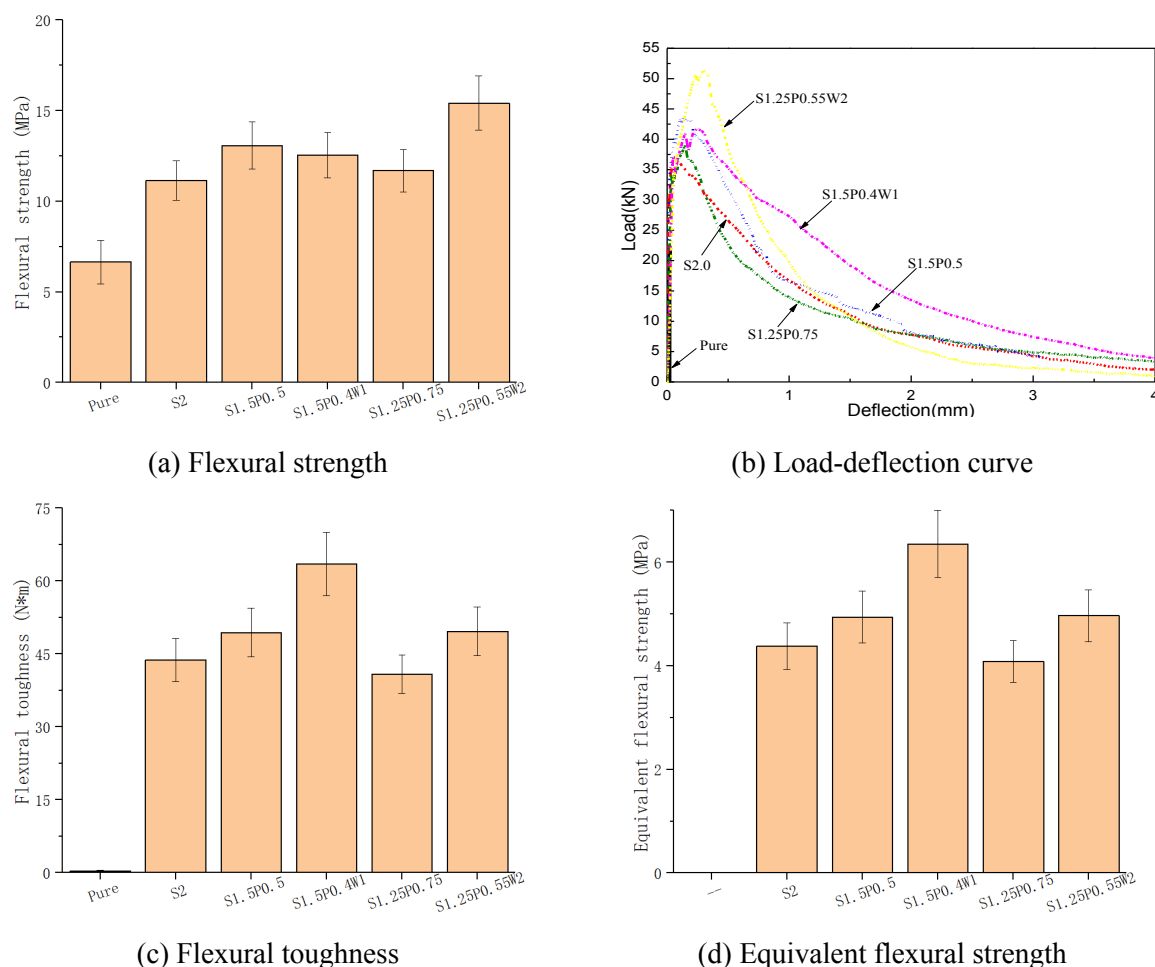


Figure 3: Flexural behaviors of various mixtures

The flexural behaviours of composites are also evaluated by the load-deflection curves shown as Figure 3(b). All fibre reinforced mortars presented an appreciable increase in the softening branch of the load-deflection curve. Generally, the steel and PVA hybrid fibre reinforced mortars presented more extended softening branch than that of S2 resulting from the positive hybrid effect of steel and PVA fibres. And the multi-scale hybrid fibre-reinforced mortars (S1.5P0.4W1) reached the most extended softening branch in all mixtures. The introduction of CaCO_3 whisker in the steel and PVA hybrid fibre reinforced mortars attained further increase in flexural response. This may result from the multi-scale design of the hybrid fibres and whisker.

The flexural toughness was employed to assess energy absorption capacity of specimens. The flexural toughness (T_b) is examined by measuring the area under the load-deflection curve up to a deflection of 3mm as presented in Figure 3(c). The addition of fibres and whisker led to notable enhancement to the flexural toughness of various mixes. Compared to mono-steel fibre reinforced mortar (S2), the steel and PVA hybrid fibre mortars (S1.5P0.5) could reach higher flexural toughness. The multi-scale HFRCC (S1.5P0.4W1 and S1.25P0.55W2) could improve flexural toughness of beams more efficiently than the conventional HFRCC (S1.5P0.5 and S1.25P0.75). This is likely result from the fibre synergy of the multi-scale system. The error bars indicate that fibres decrease the data scatters of flexural behaviours of mortar, as shown in Figure 3.

The equivalent flexural strength was employed to assess residual load-carrying ability of beams. The equivalent flexural strength is calculated by $\sigma_b = T_b L / (\delta_{tb} b h^2)$, where σ_b is the equivalent flexural strength (MPa); T_b is flexural toughness (N·m); δ_{tb} is a specific deflection value (mm, and set to 3mm here); b , h , L are width, depth and span of beam, respectively. The multi-scale HFRCC (S1.5P0.4W1 and S1.25P0.55W2) reached a greater equivalent flexural strength than the conventional HFRCC (S1.5P0.5 and S1.25P0.75), which is similar to the results of flexural toughness. This presents that the multi-scale HFRCC have a greater residual load-carrying ability than that of the conventional HFRCC.

4 Concluding remarks

A new kind of multi-scale fiber-reinforced cement-based composite, which contains 35mm hooked end steel fibre, 6mm PVA fibre and CaCO_3 whisker, was designed and tested for compressive and flexural behavior. The experimental findings, being presented in the present paper, allow to draw the main concluding remarks listed as follows.

1. From test results it turns out that, compared to the conventional hybrid fibers (steel fibre and PVA fibre), the addition of multi-scale fibres (steel fibre, PVA fibre and CaCO_3 whisker) can improve the compressive strength and toughness of cement-based composite.
2. Multi-scale fibre usage leads to notable increase to both the flexural strength and flexural toughness values of cement-based composite.
3. Based on these test results, we can conclude that there is a remarkable fiber synergy in multi-scale hybrid fibre reinforced cement-based composites with 35mm hooked end steel fibre, 6mm PVA fibre and CaCO_3 whisker.
4. It seems possible that the steel fibres and PVA fibres can be partly replaced by CaCO_3 whiskers, which is very beneficial in decreasing the production cost of fiber reinforced concrete for potential structural applications.
5. As the new HFRCC could be used to strengthen and retrofit the bridge and hydraulic structures, the durability of this new HFRCC should be carefully studied in the future.

5 Acknowledgements

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A sectional approach for the bending creep of FRC based on uniaxial tension creep tests

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Abstract

The creep behavior of FRC elements remains an important obstacle to use FRC in structural applications. Owing to the residual post-cracking strength properties of FRC, creep deformations play an important role in the cracked sections and influence durability and SLS requirements of structural elements. Therefore, it is of high importance to take creep deformations into account in the design phase.

In this paper, the results of an experimental campaign involving both bending tests and uniaxial tensile creep tests on polymeric FRC are presented. In the bending tests, a notched FRC beam is subjected to loading-unloading cycles while the deformations over the cracked section were recorded. The uniaxial tensile creep tests were performed on precracked FRC samples to quantify time-dependent crack growth.

The bending behavior of FRC can be accurately predicted by the uniaxial constitutive model of Model Code 2010 in the loading phase assuming a plane section approach. For the unloading phases, a bilinear deformation distribution is assumed and a scalar damage evolution function is fitted by an inverse analysis algorithm. The results of the sectional analysis compared favorably with the experimentally observed data.

Finally, a sectional analysis approach is developed and presented in which bending creep deformations are calculated using the uniaxial creep compliances. The initial stress and deformation distribution in the cracked section is predicted by the inverse analysis. The results show that the bending creep deformations of FRC can be quite large, and creep coefficients as high as 7 are observed within 120 days. However, it should be noted that the creep algorithm does not (yet) take into account additional cracking in time, and as such, the predicted creep deformations are a lower limit of what can be expected in reality. More research is needed to upgrade the algorithm to allow predictions including the time-dependent cracking behavior.

Keywords

Sectional analysis, creep of FRC, polymeric FRC, tensile and bending creep

1 Introduction

Fiber reinforced concrete (FRC) is a composite material in which fibers are added to the fresh concrete mix [1, 2]. These fibers are mainly used to improve the properties of the concrete in the hardened state. In structural applications, fibers can partially or totally replace the traditional reinforcement. For these purposes, the fibers provide an enhanced post-cracking tensile strength in the hardened state by bridging crack faces [3]. Commercially available fibers can be made from a number of different materials: steel, glass, synthetic and natural being the most common types [4].

Until recently, the research effort has been focusing on the properties of FRC in the fresh and hardened state [5, 6]. It has been found that the inclusion of fibers decreases plastic shrinkage cracking [7] and flowability [8] in the fresh state. In the hardened state, fibers provide a post-cracking strength to the matrix [9-11]. However, investigations on the long-term structural properties of FRC have only recently become a research focal point. The recently published Model Code 2010 [12] acknowledges that time-dependent effects may significantly alter the behavior of structural elements, both in the serviceability limit state and in the ultimate limit state. In the former, creep may cause unwanted and unacceptable deflections of FRC beams [13, 14], whilst in the latter, the residual load-bearing capacity of elements may be significantly lower than the design value based on the short-term strength assessment. However, the Model Code (MC10) does not provide design rules to take long-term behavior into account. It is clear that further research on the long-term structural properties of FRC is required.

One of these time-dependent phenomena is creep. In this paper, two topics are discussed. Firstly, the results of an experimental campaign into the creep behavior of cracked polypropylene FRC under sustained uniaxial tension loading is presented. Furthermore, the results of the creep tests were modelled in the dedicated finite-element program DIANA. Secondly, a sectional approach is presented in which the results of the uniaxial tensile creep tests can be incorporated. By using inverse analysis to determine stresses and crack widths in the cracked section of the beam, creep functions are assigned to the discretized beam section. In this way, the results of the uniaxial tensile tests can be input for a sectional approach for the bending creep of cracked FRC.

2 Experimental program

The experimental program focusses on a normal strength concrete ($f_{cm} = 43$ MPa) with 1 percentage of volume of polypropylene fibers and consists of a series of short-term characterization and long-term creep tests on three different levels: (1) single fiber [15], (2) pull-out behavior [16] and (3) FRC performance [17]. In this paper, the results of the FRC scale will be shortly presented and discussed, the details can be found in literature [17, 18].

On the FRC level, the material is firstly characterized in short-term tests, both in bending ($n = 12$) according to the European Standard EN 14651 as well as in uniaxial tension ($n = 4$) in a specifically designed tensile frame, which allows uniform crack width opening by manual eccentric load application. Furthermore, 6 beams were tested in a EN14651 three-point bending configuration with loading-unloading cycles at $CMOD = 0.1, 0.2, 0.3, 0.4$ and 0.5 mm to determine the unloading behavior. Five additional LVDTs were placed over the cracked section to measure crack opening and closing upon un- and reloading in bending. The test specimen, setup and loading speed were identical to the EN 14651 test specimen and 5 un- and reloading

cycles were considered at 0.1, 0.2, 0.3, 0.4 and 0.5 mm. A photo of the cyclic setup is shown in Figure 1, clearly showing the 5 LVDTs.

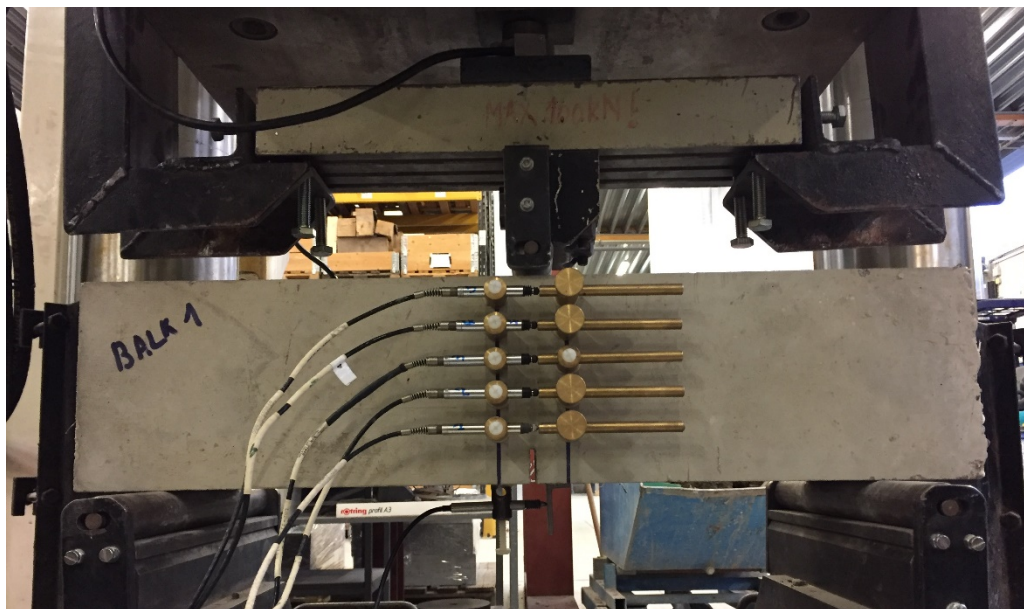


Figure 1: Photo of the cyclic bending setup with 5 additional LVDTs over the cracked section

Secondly, 14 specimens were cored from the two halves of FRC beams after flexural testing and subjected to uniaxial tensile creep loading. The cores were precracked to an initial crack width of 0.2 mm. Two different load ratios were considered, i.e. 30 % and 45 % of the residual strength at the precracking level. Of the 14 creep specimens, 7 have already been subjected to at least 6 months of sustained loading; the 7 other specimens are still running. All tensile tests are performed in a climate controlled room at a constant temperature and relative humidity of 20 °C and 60 %, respectively. In Table 1 an overview of the different tests is given.

Table 1: Number of tests specimens in the experimental program

Loading configuration	Uniaxial tension [†] (Eccentric load application)	Bending (EN 14651)
Short-term characterization	n = 4	n = 12
Cyclic loading		n = 6
Creep tests	n = 14 (7 finished, 7 running)	
- 30 %	n = 7 (4 finished, 3 running)	
- 45 %	n = 7 (3 finished, 4 running)	

[†] tests performed in a climate controlled room (20 °C, 60 %)

2.1 Bending results

The results of the EN 14651 bending tests are shown in Figure 2(a), and assuming a lognormal distribution for the determination of the characteristic values, the FRC is classified according to the Model Code 2010 [12] as a **2d** concrete.

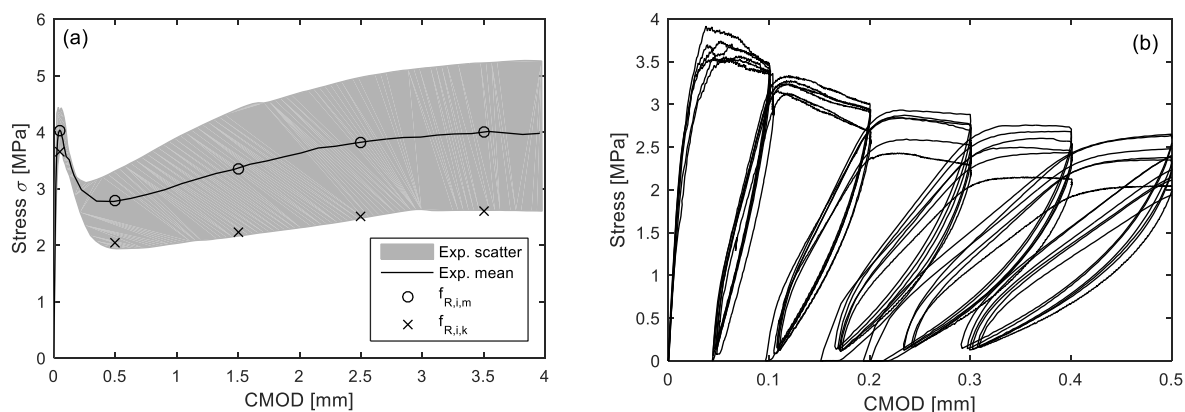


Figure 2: (a) EN 14651 bending results (b) Cyclic bending results for CMODs = 0.1 – 0.5 mm.

The results of the 6 cyclic tests on FRC are shown in Figure 2(b), only showing the CMOD until 0.5 mm as this is the area of interest. Note that the results of the cyclic tests are not included in the full bending tests and have not been taken into account in the post-cracking strength classification.

2.2 From bending to uniaxial results

The results of the 12 full EN 14651 bending test can also be used to predict the uniaxial behavior, i.e. the stress-crack width (σ -w) behavior based on equation found in the Model Code and as proposed by di Prisco et al. [19].

In the pre-peak region, equation 5.1-29 and 5.1-30 of MC10 are used with a structural characteristic length $l_{cs} = 125$ mm, as proposed in section §5.6-8 of MC10, to transform the σ - ε relation into a σ -w curve. The adopted tensile strength of the concrete is assumed to be the mean value of the tensile strength determined from the compressive strength and from the bending tensile strength, equation 5.1-3a and 5.1-8a respectively in MC10, refer to Equation 1.

$$f_t = 0.5 \times \left(0.3 \cdot f_{ck}^{\frac{2}{3}} + \frac{0.06 \cdot h^{0.7}}{1 + 0.06 \cdot h^{0.7}} \cdot f_L \right) \quad (1)$$

For the post-peak behavior, an immediate drop in strength is observed in accordance with the MC10 equation 5.6-10, based on the fracture energy of the concrete matrix. Following this drop in strength, the approach proposed in [19] is adopted, relating the residual strength parameters f_{R1} and f_{R3} to the uniaxial tensile behavior at crack width $w = 0.5$ and 2.5 mm.

In this way, a multi-linear constitutive σ -w relationship is calculated based on the EN 14651 bending results and the compressive strength of concrete. The results of this approach are shown in the following section, in comparison with the experimentally obtained results.

2.3 Uniaxial results

In uniaxial tension, 4 specimens are tested until a crack width of 4 mm was recorded and 14 specimens have been precracked to an initial crack width of 0.2 mm. The results of all 18 uniaxial tension tests are shown in Figure 3(a), and highlighted to a crack width of 0.3 mm in Figure 3(b). Note the unloading branch for the 14 creep specimens at $w = 0.2$ mm.

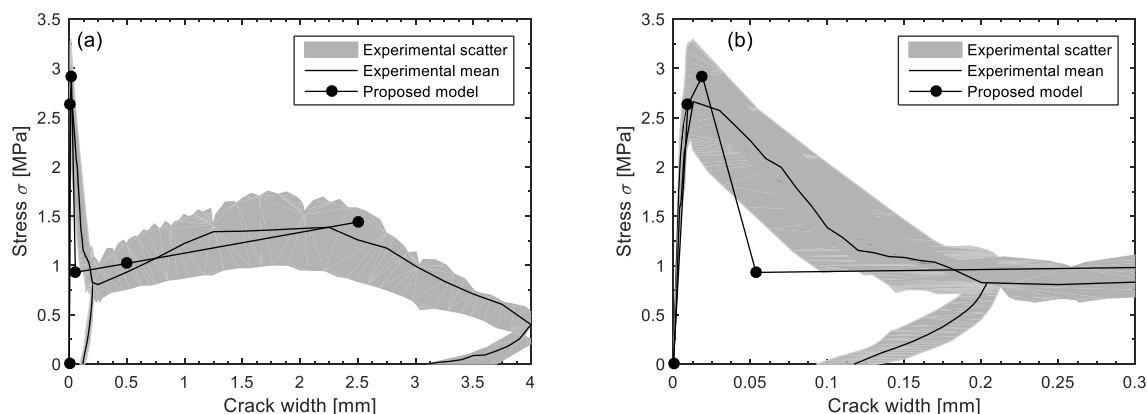


Figure 3: Uniaxial results together with the proposed model for (a) the whole test (b) until $w = 0.3$ mm

From Figure 3, it is clear that the combined MC10-[19] model is able to accurately describe the uniaxial behavior based on bending results. The model however predicts a larger strength loss in the immediate post-peak region, but this can be attributed to the relatively low stiffness of the tensile frame which cannot absorb the energy release associated with matrix cracking.

Following the precracking, the samples are subjected to creep tensile loading at two different load ratios, i.e. 30 % and 45 %. In this paper, the results for the 7 completed creep tests are shown in the next section together with the numerically obtained creep deformations.

3 Numerical analysis

The finite element analysis aims to describe and predict the creep behavior in uniaxial tension based on the results of the single fiber and pull-out behavior, using a discrete treatment of the fibers. The details of the model characteristics and calibration are given elsewhere [20]. In Figure 4 the results of the numerical analysis are shown together with the experimentally observed creep deformations. Note that the general trend is quite accurately captured, but the initial deformations upon loading are overestimated by the numerical model. This is attributed to low fiber stiffness in the numerical model which is calibrated on the secant modulus defined in EN 14889-2. However, since the load application in the FEM is instantaneous, the tangent modulus is better suited to capture the initial deformation at the instantaneous load application. Nevertheless, the creep deformations as obtained from the FEA are retained.

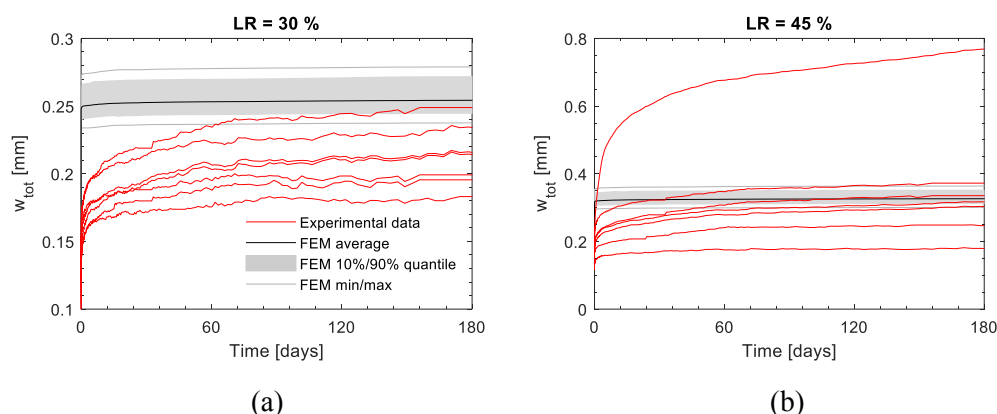


Figure 4: Comparison of the creep deformations of the experiments and the numerical model for (a) 30 % and (b) 45 %

4 Sectional analysis

Even though FEA is a very powerful tool, in most real design problems, it is quite often prohibitively expensive to construct, run and analyze the results. Therefore, most design approaches consider a sectional approach which can be easily understood and adopted. In this section, a proposal for such a sectional approach is given to take creep deformation into account. The basis of this approach is a discretized cracked section subjected to bending moments. Uniaxial creep deformations are then assigned to the discretized elements in the cracked section.

4.1 Loading path – plane section

Before creep deformations can be taken into account, the first step is to calculate the stresses in the cracked section, both upon loading (i.e. following the EN 14651 test) and unloading (i.e. at the beginning of the creep test). Since the σ - w constitutive relation is known, it is possible to calculate the stresses upon loading, assuming a plane cracked section, ensuring horizontal equilibrium is fulfilled. The resulting bending moment can then be compared against the experimentally observed. The results of the calculation are shown in Figure 5(a) until CMOD = 4 mm and in Figure 5(b) until CMOD = 0.5 mm. The approach is able to quite accurately capture the test results, albeit it overestimates the tensile strength of the concrete somewhat. Except near the bending tensile strength, the sectional predictions remains within the experimental bounds for the entire test.

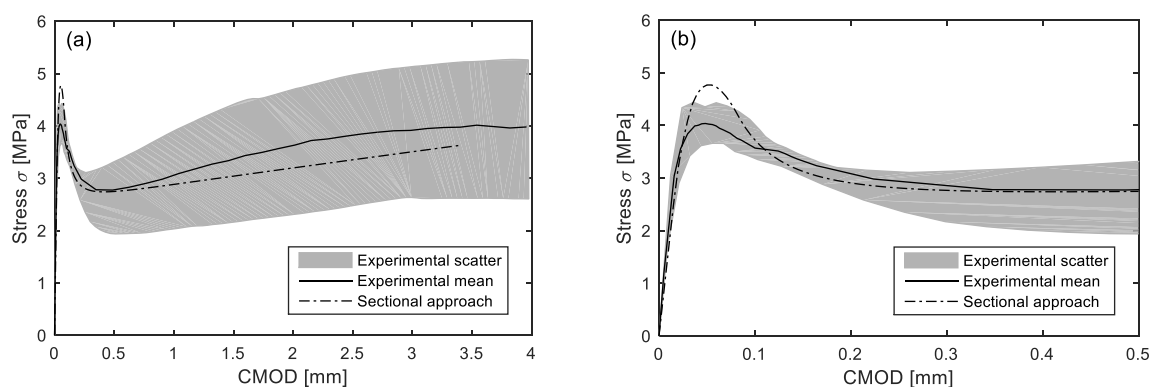


Figure 5: Comparison of the sectional approach and the experimental results for (a) complete test and (b) until 0.5 mm

4.2 Unloading path – bilinear

Upon unloading, the plane section approach cannot be assumed correct anymore, as unloading under such an assumption would entail complete recovery of crack widths, which is not observed experimentally. Therefore, a bilinear deformation distribution over the section height is proposed with the inflexion point at the height of the undamaged zone. The bilinear distribution is calibrated on the unloading paths of the FRC bending tests, refer to Figure 2(b). The inputs for the inverse analysis are the CMOD values and the applied force, and the algorithm finds the scalar damage evolution $D(w)$ as a function of the total deformation w . The optimized parameter is the sum of the squared force ΔH and moment error ΔM . The inverse analysis algorithm for the unloading is summarized below:

Given $F_{ext,i}$ and $CMOD_i$ for all unloading points i ,

Find $D(w)$ in a bilinear deformation distribution,

Such that $\Delta E = \sqrt{\Delta H^2 + \Delta M^2}$ is minimized,

With
$$\begin{cases} \Delta H = b \cdot \Sigma \sigma_j \\ \Delta M = (b \cdot \Sigma (y_j \cdot \sigma_j) - M_{ext}) / M_{ext} \end{cases}$$

for all elements j over the height of the section

The results of the model are quite accurate as 99 % of all unloading points i have an error ΔH and ΔM between ± 5 N and ± 1.5 %, respectively. Note that the experimentally observed deformations upon unloading over the height of the section are not used (refer to Figure 1), and as such, these experimental data points can be used to validate the model. The comparison between the model and experimental data is shown in Figure 6, for the full load (100 %) and at a load ratio of 50 % at unloading from different CMOD values (0.1 – 0.5 mm).

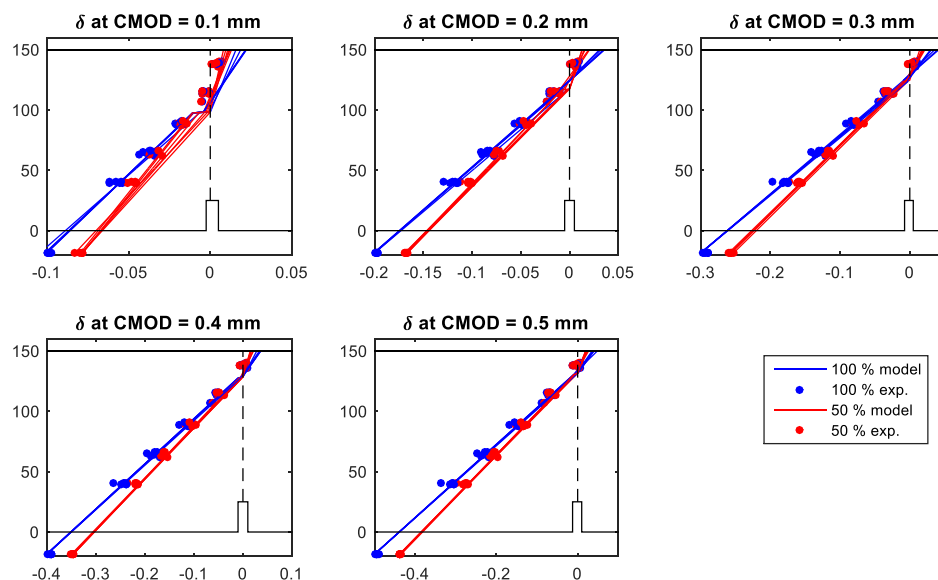


Figure 6: Deformation profiles predicted by the model in comparison with experimental data

The lines show the deformation predicted by the model, while the dots indicate experimental data. Note that at full load, the bilinear model predicts a linear deformation profile, which is

expected as shown in section 4.1. At lower loads, the linear profile quickly becomes bilinear in order to achieve residual deformations. As stated, the experimental deformations (dots in the figure) are not used in the inverse analysis but show a good agreement with the predicted model deformations. The scalar damage evolution is shown in Figure 7(a), indicating that damage accumulates very quickly. The damage evolution is calculated independently for every unloading step, but the results show that all curves are in relatively close agreement with each other. As an example, the stress distribution over the sectional height is shown in Figure 7(b) upon unloading from 0.5 mm for the full load (100 %) and at 50 %.

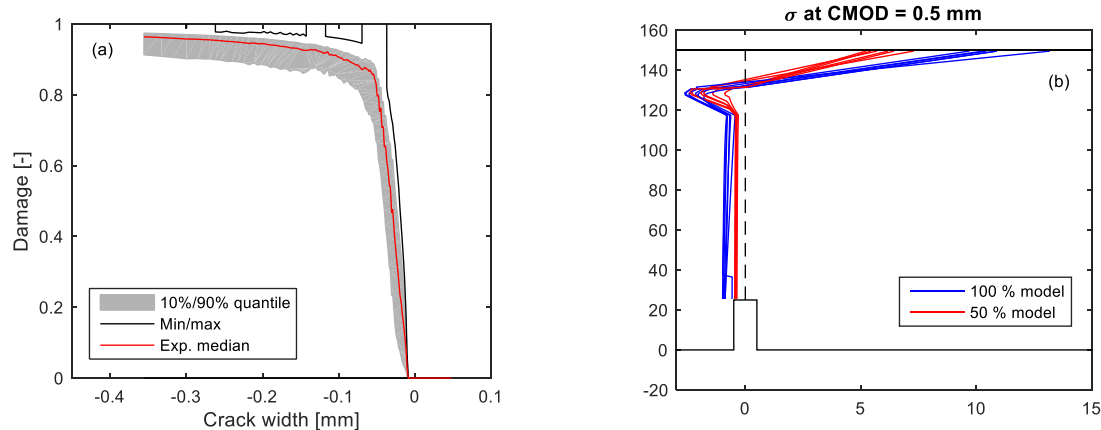


Figure 7: (a) scalar damage evolution for all specimens (b) stress profiles for 100 % and 50 %

4.3 Creep implementation

The deformation and stress profile shown in Figure 6 and Figure 7(b) at 50 % load after unloading from CMOD = 0.5 mm can be used as a starting point to predict bending creep based on the uniaxial creep compliance functions. It is assumed that the undamaged tensile creep behavior is identical to the undamaged compressive creep, and that the damaged tensile creep compliance is independent of the initial crack width. Under these assumptions, the creep deformation of an FRC beam under bending can be calculated, and it is shown in Figure 8(a) and (b) in terms of an increase in CMOD and the creep coefficient ϕ (defined as $CMOD(t)/CMOD_0$) respectively.

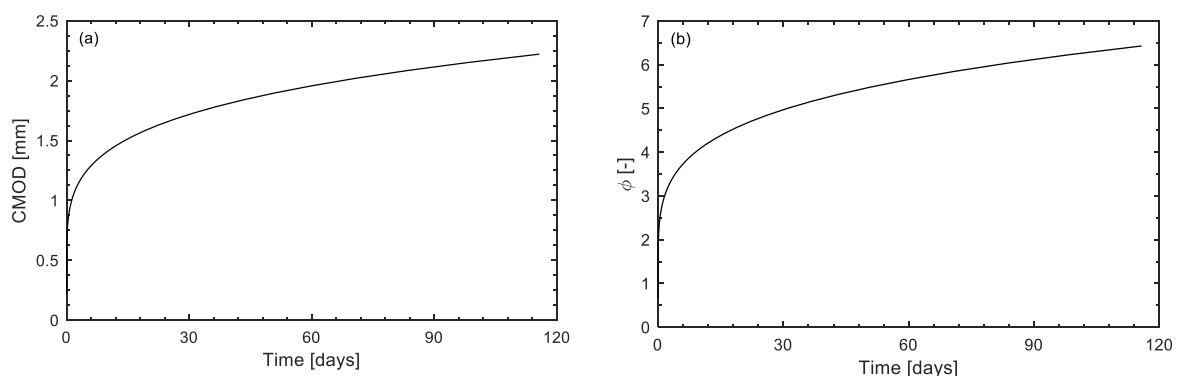


Figure 8: Increase in (a) CMOD and (b) creep coefficient for the bending test at 50 %

While these results provide an indication of the expected increase in deformations, it should be noted that in the real beam, a stress redistribution will take place owing to the increase of the crack height, a smaller compressed zone and higher associated compressive stresses, further increasing the creep deformations. This needs further investigation, both on an experimental level as well as in the sectional approach. Nevertheless, the results presented here can serve as a first indication and, as such, are a lower limit of the real deformations that can be expected. As has been shown, these CMOD increases are significant and should be taken into account where SLS requirements are of importance.

5 Conclusion

In this paper, the results of an experimental campaign involving bending tests and uniaxial tensile creep tests are presented. In the bending tests, a cyclic behavior of loading – unloading cycles is imposed to investigate residual crack widths and the unloading performance.

Furthermore, a sectional approach is presented that is able to accurately predict and describe the deformations and stresses in the cracked section at the loading and unloading cycles. In the former case, a linear deformation distribution is sufficient to describe the behavior. For the latter cycles, a bilinear distribution is needed to predict the residual crack widths upon unloading. A scalar damage function is fitted in an inverse analysis to come to the results, which compare favorably with the experimental data.

Lastly, the results of the stress and deformation profiles is used to predict bending creep based on the uniaxial creep behavior of undamaged and cracked FRC. While the approach presented here predicts reasonable results, it should be noted that the creep prediction does not take time-dependent cracking into account. As such, the predicted creep deformations with creep coefficients exceeding 7 after 120 days should be regarded as the lower limit of the real deformations. More research is needed to update the creep calculations to include all effects. Nevertheless, the approach presented here provides interesting insights into the stress distribution in cracked sections of FRC.

6 Acknowledgements

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Optimized quality control procedure with the Barcelona test and the inductive method for FRC

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Abstract

The optimization of resources and materials with the objective of cutting costs has a great relevance at all stages of the design of a structure, including the quality control. In fibre-reinforced concrete (FRC), the three-point bending tests are performed routinely as means of quality control to determine the residual strength. Nevertheless, the high scatter of its results and its complex setup requiring special equipment encourages the use of alternative tests to determine the post-peak strength of FRC. In this respect, the Barcelona test arises as an alternative method to characterize FRC due to a more reduced variability of the results, a lower concrete consumption and the simplicity of its execution. Moreover, the inherent heterogeneity of FRC directly affects the content of fibers within the concrete, which is one of the main factors governing the post-cracking strength of FRC. In this regard, the inductive method is proposed as a complementary test to assess the content of fibres during quality control using the same specimens to be tested under the Barcelona test. Therefore, the main aim of this study is to present a methodology to correlate the three-point bending test with the Barcelona test and show the feasibility of using the latter as an alternative method. Furthermore, the combination of the Barcelona test with the inductive method leads to a more complete characterisation of FRC.

Keywords

FRC, quality control, bending test, Barcelona test, inductive method, content of fibres

1 Introduction

The three-point bending test is one of the most extended methods to determine the post-cracking strength of FRC. This test is extensively used for design given that the geometry of the specimens and the Mode I fracture crack propagation are representative of the real behaviour of an element subjected to flexural stresses. However, as suggested by the Belgian standard NBN B 15-238, the three-point bending test is not a suitable method for the systematic quality control of FRC. This is mainly due to the high coefficient of variation, which can reach values above 20% [1,2]. Additionally, the complex setup of the test requiring big-sized samples, high precision equipments and a closed-loop configuration hinder the execution of the test and increase the costs.

Alternative methods can therefore be used in an attempt to overcome the drawbacks of the three-point bending test. In this context, and according to the *fib* Model Code 2010 [3], the use of alternative tests can be accepted if correlation factors are proven. In this regard, the Barcelona test [2] arises as an alternative to streamline the quality control given its simpler setup and lower scatter [4]. In order to meet project requirements, a minimum strength of the material is usually specified in quality control. If the test results do not reach this minimum value, the material must be discarded and a new batch of concrete must be manufactured. In this sense, the mere fact of changing the test methodology would reduce the scatter as well as the amount of material rejected and, hence, the costs while meeting the acceptability criteria.

Given the influence of the content of fibres on the post-cracking strength of FRC, the standard EN 14721 describes the procedure to perform the control of the content of steel fibres in FRC. According to this, fresh concrete needs to be washed and hardened concrete samples need to be crushed to separate the fibres. This results in a highly time-consuming procedure to determine the content of fibres thus limiting the number of samples that can be tested. For this, an alternative test such as the inductive method [5] is here presented. It consists of a non-destructive test that may be conducted within less than a minute providing the content of fibres through a calibrated relation between the inductance and the content of fibres.

Even in the case of different cracking mechanisms, the use of correlations between concrete properties measured through different test methods is a common and widely accepted procedure. For example, there are several equations available to transform the indirect tensile strength measured in the Brazilian test into the indirect flexural strength measured with bending tests [6]. In the case of FRC, Minelli et al. [7] already proposed a correlation between the Round Panel Test and the UNI flexural test based on the energy released and the residual strength.

The main objective of this document is to describe the methodology to correlate the results of the Barcelona test and the bending test for quality control given a concrete mix and type of fibre. Moreover, the procedure to determine the content of fibres using the inductive test and the method for its calibration is also presented. The correlation proposed represents a tool towards a simpler, faster and less expensive quality control of FRC based on the Barcelona test (BCN). This approach is in line with the *fib* Model Code 2010 and may also serve as an example for future correlations with other tests applied to FRC.

2 Experimental program

2.1 Specimens and tests

Three batches of the same FRC mix with a reinforcement of 30 kg/m^3 of steel fibres were produced to be tested under the three-point bending test and the Barcelona test. For this, prismatic beams with dimensions of $150 \times 150 \times 600 \text{ mm}$ and cubic specimens of $150 \times 150 \text{ mm}$ were casted for the flexural test and the Barcelona test, respectively. For batch 1 (B1), 10 prismatic and 20 cubic specimens were casted. For batches 2 and 3 (B2 and B3), 5 prismatic beams and 10 cubic samples were produced per each batch.

The bending test was conducted according with EN 14651 with a notch at the mid-span of each beam, thus weakening the central section of the specimen where the load was applied. A CMOD (Crack Mouth Opening Displacement) transducer was placed at the notch between two steel sheets (see Figure 1) to simultaneously record the crack opening and the load with a closed-loop configuration. The test is controlled through the CMOD at a rate of 0.05 mm/min until the crack opening reached 0.10 mm . Afterwards, the speed was increased to 0.20 mm/min until a crack opening of 4 mm was achieved, point at which the test was considered to be totally performed.

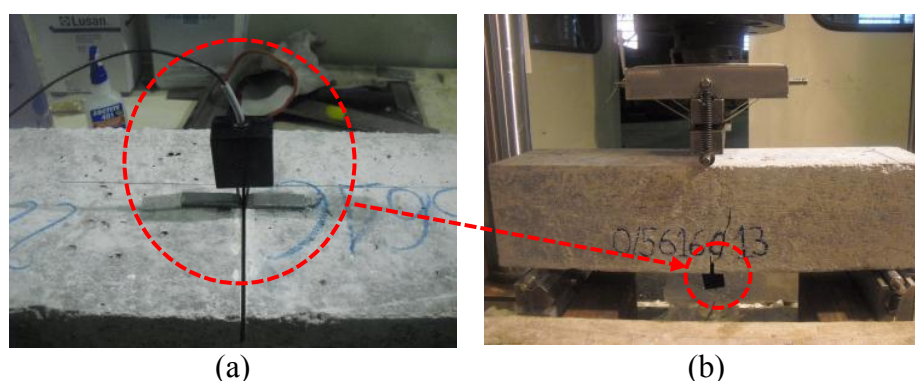


Figure 1: CMOD (a) and general overview of the three-point bending test (b)

The inductive method may be used to determine both the orientation and the content of fibres in either cubic or cylindrical samples. The inductive test is a recently developed method to characterize steel fibre reinforced concrete [5,8]. For such a test, the specimens must be placed within a plastic container with an established cylindrical geometry as shown in Figure 2. Copper or aluminum wire coils rolled around the container constitute the sensor element of the system. A small electrical current goes through the coil and produces a magnetic field around the device, interacting with the steel fibres inside the concrete and providing the inductance as data output. Based on the equations developed in previous research [8,9], the orientation number and the contribution of the fibres in each direction may be assessed.



Figure 2: Inductive method equipment

After conducting the inductive test in the cubic samples of the three batches, the Barcelona test was performed according to the procedure described in UNE 83515 and as depicted in Figure 3. Two steel cylindrical punches are placed concentrically above and below the cubic specimen. These punches have a diameter of $\frac{1}{4}$ of the diameter of the inscribed cylinder in the cubic sample and a height of 25 mm. The hydraulic press transmits a load through the punches to the sample at a constant rate of 0.5 mm per minute. The compression load applied onto the punches produces a conical triaxial stress state from the centre to the edges of the specimen. This results in an increase of the diameter of the sample and, thus, in concrete cracking when the peak tensile strength is reached leading to the activation of the fibres.

Based on the standard UNE 83515, the crack opening is measured through the Total Circumferential Opening Displacement (TCOD). However, previous research [10] describes an analytical correlation between the TCOD and the vertical displacement, this leading to avoid the necessity of using the TCOD transducer while replacing cylindrical specimens for cubic ones. In this paper, the approach based on the vertical displacement is used for determining the residual strength with the Barcelona test.

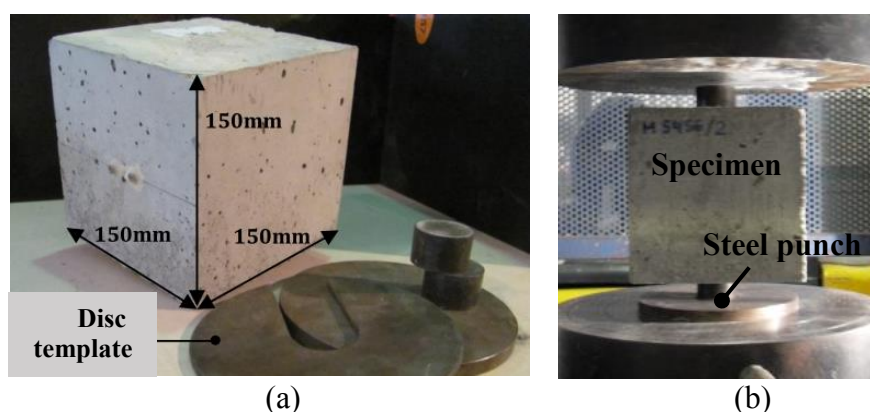


Figure 3: Cubic specimen and punches (a); test setup (b)

The content of fibres inside the specimen can be estimated based on a correlation between the real content of fibres of the sample and the inductance measurements obtained through the inductive test. To calibrate such correlation according with the type of fibre used in this experimental program, 6 cubic specimens were crushed after having performed the Barcelona test. The fibres of these specimens were separated from concrete and weighted to determine the content. This content was subsequently correlated with the inductance measurements of the inductive test at the three perpendicular directions of the specimen.

3 Results

3.1 Post-cracking strength

The average results of the three-point bending tests conducted on the three batches are presented in Table 1. The residual strengths at crack openings of 0.5, 1.5, 2.5 and 3.5 mm (f_{R1} , f_{R2} , f_{R3} and f_{R4} , respectively) are shown, along with the coefficient of variation (CV) of each parameter. The results of the residual strength of cubic specimens tested under the Barcelona test and determined as described in the standard UNE 83515 are shown in

Table 2. The results show the average residual strength for vertical displacements of 0.5, 1.5, 2.5 and 3.5 mm ($f_{R0.5}$, $f_{R1.5}$, $f_{R2.5}$ and $f_{R3.5}$, respectively), as well as the scatter associated to each strength.

Table 1: Bending test results of B1, B2 and B3

Parameter	B1		B2		B3		B1-B2-B3	
	Average	CV %	Average	CV %	Average	CV %	Average	CV %
f_{R1} [MPa]	2.36	22.2%	3.10	23.6%	3.02	23.9%	2.83	14.4 %
f_{R2} [MPa]	2.49	26.1%	3.06	24.8%	3.17	25.6%	2.91	12.7 %
f_{R3} [MPa]	2.39	26.9%	2.83	22.1%	3.03	23.8%	2.75	11.8 %
f_{R4} [MPa]	2.27	27.5%	2.60	23.5%	2.80	23.2%	2.56	10.4 %

Table 2: Barcelona test result of B1, B2 and B3

Parameter	B1		B2		B3		B1-B2-B3	
	Average	CV %	Average	CV %	Average	CV %	Average	CV %
$f_{R0.5}$ [MPa]	1.48	14.0%	1.54	9.1%	1.47	23.3%	1.50	2.5 %
$f_{R1.5}$ [MPa]	1.05	17.7%	1.02	12.0%	0.98	22.6%	1.02	3.5 %
$f_{R2.5}$ [MPa]	0.86	19.0%	0.75	19.0%	0.80	20.8%	0.80	6.9 %
$f_{R3.5}$ [MPa]	0.74	21.2%	0.60	24.6%	0.68	21.3%	0.67	10.4 %

The results show that Batch 1 presented the lowest residual strengths when conducting the three-point bending test. With the exception of f_{R1} , B1 also presented the highest scatter among the three batches, even though the dispersion was also similarly high in B2 and B3 with CV around 23% and 25%. Conversely, the results of the Barcelona test present the highest residual strengths in B1, with the exception of $f_{R0.5}$. Notice that lower residual strengths when compared to the three-point bending test are achieved, which is mainly attributed to the different cracking mechanism and the higher cracking area.

Regardless of the different results among batches, the average strengths (B1-B2-B3) in both the bending and the Barcelona test present a more reduced scatter. Considering the three batches together, for the bending test the CV is reduced to a value around 12%, whereas in the Barcelona test the average scatter is 5.8%. The similarity between the mechanical strength of the different batches reflects the homogeneity of the material, thus revealing a properly manufacturing procedure.

3.2 Correlation between tests

In order to achieve a correlation between the mechanical parameters of both tests, it is necessary to determine an equation using the experimental results obtained through the three-point bending test and the Barcelona test. A parametric analysis yielded that if using the load and the energy of the Barcelona test at displacements of 0.5, 1.5, 2.5 and 3.5 mm it is possible to obtain a correlated value of the load of the bending test at crack openings of 0.5, 1.5, 2.5 and 3.5 mm. Each crack opening of the bending test is correlated with the displacement counterpart of the Barcelona test in Equation 1. In other words, to obtain the flexural load at a CMOD of 0.5 mm ($F_{3PBT,0.5}$), the load and the energy of the Barcelona test at displacements 0.5 mm ($F_{BCN,0.5}$ and $E_{BCN,0.5}$) needs to be used.

$$F_{3PBT,i} = a \cdot F_{BCN,i} + b \cdot E_{BCN,i}^2 \quad (1)$$

At a first stage, it is necessary to count with results of both the bending and the Barcelona test so the parameters a and b of Eq. 1 can be calibrated given that both values depend on the properties of the concrete and the fibres and need to be recalibrated under any mixture of fibre modification. In this regard, the experimental results of B1 and B2 were used for the calibration, while B3 was used to verify the results. For this, instead of using only the average values of B1 and B2, the results of all the specimens tested were used in order to have a higher amount of data and increase the reliability of the correlation with more accurate results of a and b .

Notice that B1 presented 10 specimens for the three-point bending test and 20 for the Barcelona test. The results of the Barcelona test were therefore averaged by pairs, thus obtaining 10 results to be correlated with the 10 results of the bending test. The same procedure was followed for B2, which presented 6 specimens for bending and 12 for the Barcelona test. The parameters a and b after the calibration are shown in Table 3 for each crack opening.

Table 3: Calibrated parameters of correlation

CMOD [mm]	a	b
0.5	$1.19 \cdot 10^{-1}$	$7.11 \cdot 10^{-4}$
1.5	$1.76 \cdot 10^{-1}$	$1.80 \cdot 10^{-4}$
2.5	$1.76 \cdot 10^{-1}$	$1.53 \cdot 10^{-4}$
3.5	$1.95 \cdot 10^{-1}$	$8.94 \cdot 10^{-5}$

A schematic representation of how to obtain the load and the energy parameters of the bending and the Barcelona test is shown in Figure 4. The energy of the Barcelona test is calculated as the area under the Load-displacement curve up to the selected displacement. The example represents the parameters for a CMOD or a vertical displacement of 0.5 mm, even though the rest of parameters are calculated similarly.